02 May 2013, 4:00 pm - 6:00 pm

Technical Analyses and Lessons of The Embankment Failure at the Ajka Red Mud Reservoir

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Commemorate the Legacy of Ralph B. Peck
April 29 - May 4, 2013 CHICAGO
Seventh International Conference on Case Histories in Geotechnical Engineering
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TECHNICAL ANALYSES AND LESSONS OF THE EMBANKMENT FAILURE AT THE AJKA RED MUD RESERVOIR

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ABSTRACT

This presentation aims at providing an informative fact-based description of the complex reasons resulting in dam ruptures. The investigations focus on the tragic damage of red mud reservoir No. 10 in Ajka and restrict itself to the engineering aspects of the damage. The analysis of the direct and indirect reasons for the tragic incident may serve as a lesson to our engineers and specialists as well as provide guidelines and revelations with the intention of improvement while emphasizing the necessity of deeper comprehension. Neither this presentation nor the engineering analyses aimed at the search for scapegoats behind the failures.

INTRODUCTION

In Hungary three alumina plants have been built. One of them in the vicinity of Ajka town has been operational since 1942, and collected red mud in a reservoir in the valley of the Torna stream. The product of alumina production, 15.7 million m³ of red mud was deposited in 10 reservoirs in the valley of the Torna stream. The red mud is a waste product of the Bayer process. Bauxite is crushed and ground in mills and heated; alumina is precipitated by washing the bauxite with a hot solution of sodium hydroxide (NaOH) under pressure.

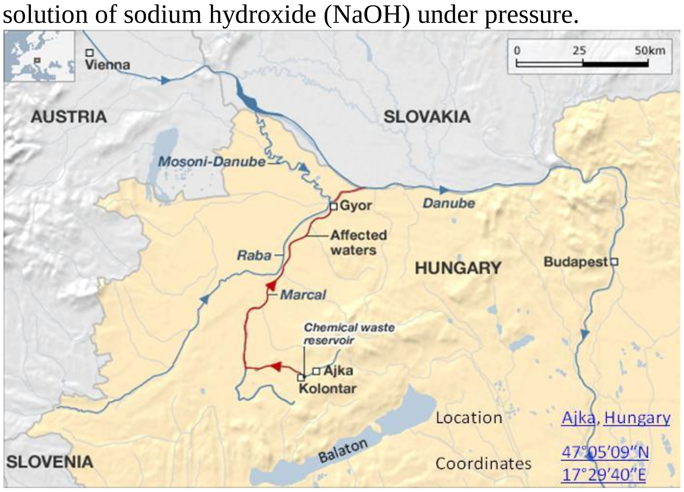

Fig. 1. The site and affected localities (http://www.bbc.co.uk/news/world-europe-11491412)

Around 24-45 percent of red mud is ferrous oxide (responsible for its reddish-brown color), but it also contains other metallic compounds. Red mud is not poisonous, but it is a hazardous material, due to its sodium hydroxide content. This highly alkaline (pH 12-13) material was transported by pipeline to reservoirs.

On October 4, 2010 at 12:25 the northwestern part of the dam of red mud reservoir No. 10 has collapsed, and near one million m^3 alkaline red mud mixed with water has plunged in the valley of the Torna stream. Fig. 1.–Fig5. The red mud flooded the valley-side parts of the village Kolontár and the town Devecser, then trough the Marcal river it has reached the river Danube in very short time. 10 people died, 123 injured, 260 houses became uninhabitable, and significant ecological damage occurred.

The presentation is based on a series of expert tests performed to examine the stability aspects of the embankments of reservoirs as required by the authorities (Mecsi, J. 2010-2012). The analysis shall not aim to examine the complex issue of deficiencies in plant operation or in the earlier management of the reservoir, neither shall it consider the measures to have been taken by the operating company or the authorities in order to minimize or to help recognize the risk of a catastrophic dam failure.

Fig. 2. The corner of the North and the West with water extension dam (photo: Sándor H. Szabó, MTI)

Fig. 3. The west and the north dam after the failure (index.hu, photo: AFP - Samuel Kubani)

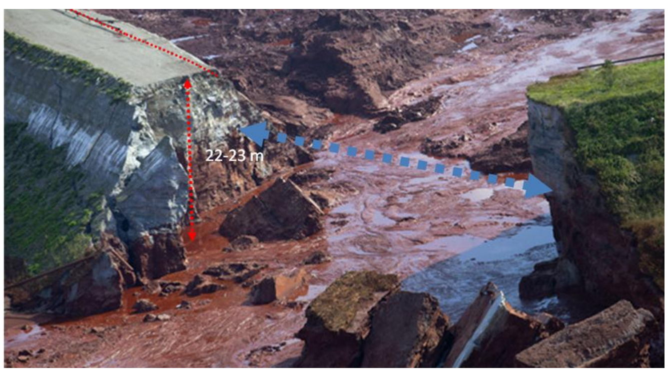

Fig. 4. Aerial photo with the alkaline red mud mixed with water extension

Fig. 5. Natural-color satellite image of the area surrounding the spill(http://redsludge.bm.hu/wp-content/uploads/2010/10/image002.jpg)

Fig. 5. Aerial photo with the flooded territory of Kolontár (photo: MTI)

2.GENERAL CONDITIONS

The embankment of the reservoir in question must be examined as part of the entire reservoir system, as the different parts of the system have a mutual impact on each another. The mutual impact is primarily caused by the water pressure and the pressure exerted by the tailings, these constituting the largest part of the load. We must consider the fact that the entire reservoir system had been built on an extensive gravel terrace covered by a mixed composition of mud-clay-sand deposit of the periodical water flow of the 4-5 m Torna stream bed.

The tragic dam failure of the north-western corner of reservoir No. 10 highlighted the importance of a complex analysis.

Important aspects of the study are:

- the geological-geotechnical conditions and the morphological properties of the area;
- the conditions changed as a result of favorable technical interventions or technical interventions believed to be favorable;
- the extreme weather, precipitation and dynamic wind load conditions;
- the specific features of the substance stored in the reservoirs;
- the resistance reducing effects of pressures due to the transition of the substance from a more favorable condition to an unfavorable state of liquefaction;
- the different rigidity properties of the unfavorable embankment connections;
- and several other factors that could not have been considered by earlier regulatory systems.

Figure 7 shows an aerial photo of the interconnected system of reservoirs in its condition prior to the embankment failure.

Fig. 7. Overhead view of the reservoir system in an aerial photo of June 2010

The present study does not aim to identify scapegoats of the incident. The findings and conclusions derived from the examinations may be further refined and supplemented in the future in light of further facts and data yet to be revealed.

3. HYDROGRAPHICAL-HIDROGEOLOGY CONDITIONS

The industrial plants and the red mud reservoirs are located in the 1-3 km wide eroded valley of the Torna stream. Figure 8 shows the original morphology on the second military survey map (1852). The territory had a basin-like character, fill wit course gravel and sand and cover fine sand and silt, hence is had a swampy character, collected the water. Before reaching the town the stream flows in a regulated bed. The original streambed was below the red mud reservoirs No. 8-10, but the stream was diverted in the 1990s when the reservoirs were built. The damaged dam of red mud reservoir No. 10 had been built on swampy ground. Geo-morphologically they are defined by series of mountain margin Mesozoic horsts and bench lands with piedmont surfaces and mountain margin sedimentary deposits.
Figure 8 shows the reservoir system after the failure by the aerial photo. From the picture you can see visually the reservoir system and the relationship with surrounding terrain. (Schweitzer F. et al)

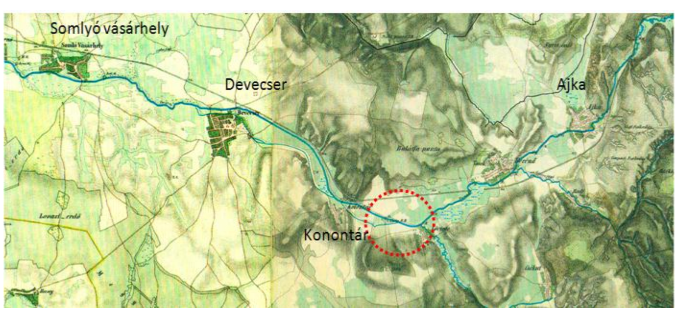

Fig. 8. The site before the construction of reservoirs on the second military survey map (1852)

Fig. 9. Picture and altitude figures of the reservoir system after the embankment failure in October 2010 (http://redsludge.bm.hu)

Figure 10 shows the site situation in the years 1970-1976. From No.1 to No.9. reservoirs were already in operation, most of the finished the filling with red-mud material, but the reservoir No. 10 was still in the planning stages. The bed of the Torna creek position was near to the west side of the dam. The idea was that reservoir area further expand to the northern part, but that did not materialize.

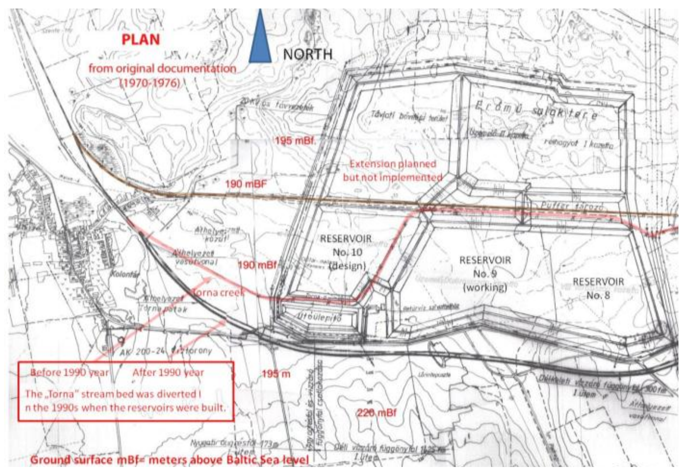

Fig. 10. The morphological map from the original documentation before the built the reservoir No. 10. The Torna stream's bed position changes when the dams of the reservoir were built

4. SUBSOIL CONDITIONS

Figure 11 shows the subsoil conditions of the area based on detailed soil tests performed between 1975 and 1980.
The reservoir system was built on the gravel terrace ("gravel basin"), in the 1-3 km wide eroded valley of the Torna stream. The gravel terrace 5-6 m deep sedimentary deposit.
The deeper part of the "gravel basin" consist of the very permeable gravel with water permeability of k= 10^{-1} cm/sec.

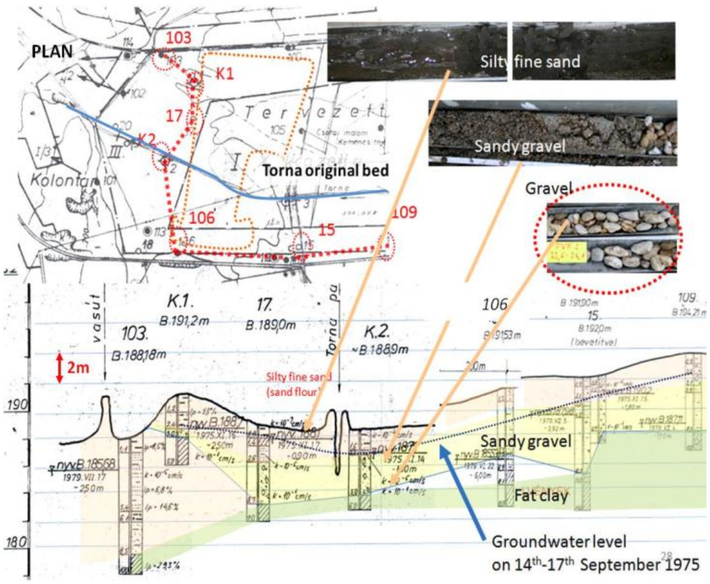

Fig. 11. Illustration of the subsoil conditions of the reservoir system based on the original plans (1975 year). (ny- groundwater level at rest ;k- coefficient of permeability; Ip- Index of plasticity)

The gravel terraces are not homogeneous. This is illustrated by an exploratory borehole in different soil depths in soil samples analyzed in terms of particle size distribution curves. (Fig. 12) The geological origin, the gravel content increases with depth, diameter of the particles is not evenly spread, dominant in the fine sand of the same particle size range, which shows the tendency of soil potential liquefaction.

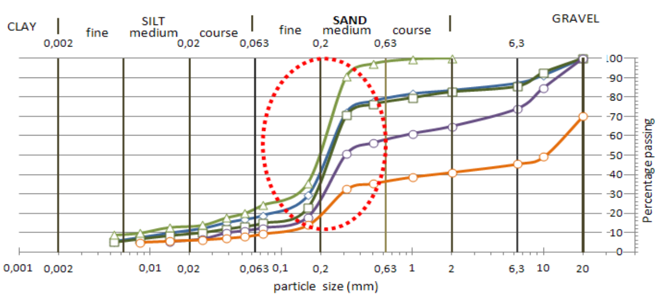

Fig. 12. Typical particle size distribution diagrams of the soil

In the year 1989 soil excavations and geophysical measurements were conducted at the leg of reservoir No. 10.'s dam, and the soil profiles were recorded. Figure 13 shows the soil profiles near the north-west corner of the reservoir No. 10, and the photo illustrates the testing locations. It is important to remark that near the north-west corner of reservoir No. 10 sand-silt is to be found (the former Hungarian naming standards "flour sand" soil) and sandy-gravel layers above the fat clay. This "sand-floor" layer was 1.2 to 2.4 m thick.

The storage space of such other part a similar unfavorable soil layer were not detected.

On the right upper part of the Figure illustrates when 2011. in November created a drainage ditch, highlighting the liquefiable -unstable soil conditions. The trench sidewalls suddenly of excavation collapsed.

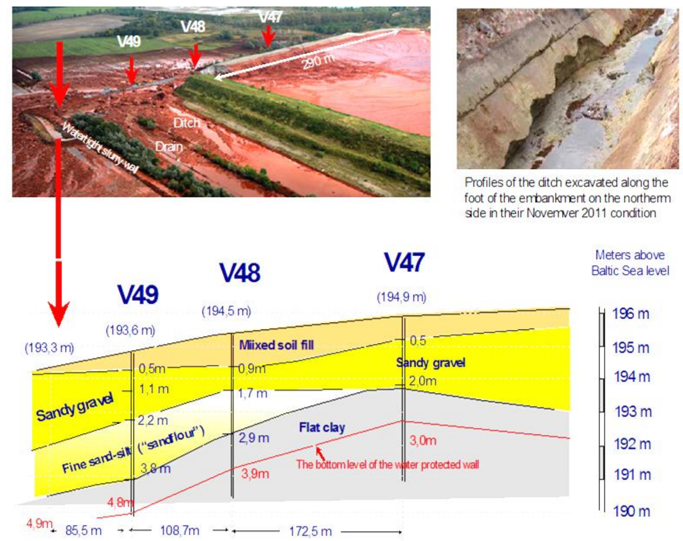

Fig. 13. Soil profile near the West –North corner of the reservoir No. 10. The top right side of the figure shows the position of the soil investigations

This soil layer unfavorable is that in fine sandy-silt fractions may move under higher water pressure conditions. As a result, grain size becomes coarser on the inner side, while a „clay plug" may develop towards the edge of areas in motion. Such a clay plug is characterized by a sudden movement under a significant change in pressure and consequently a mudflow-like grain movement may evolve. Such a process may be extremely fast and unexpected.

Figure 14 shows the typical results of the soil investigations by the boring logs after the failure. Compare with the earlier investigation results you can see that the instable fine sand-silt layer change to the red mud layer.

It will readily be conceived that the characteristics of the subsoil underlying the hydraulically solidified slag–ash embankment constitute an important aspect of the stability of embankments. It is also important to examine whether the subsoil may become liquefied under larger water pressures

The CPTu static penetration tests performed on October 7, 2010 in the direct vicinity of the damage location did not show any resistance until a depth of 2 m, below which it indicated a sandy gravel layer of a thickness of 3 m. Fig.15. The value of peak resistance was 40-50 Mpa, which indicates a compacted condition. An extremely high water pressure value of su=500-750 kPa was measured in the water bearing layers of sandy gravel and gravel, which equals to the water pressure of a 50-70(!) m water column. Note: the order of magnitude of the measured value is significant even considering the fact that pore water pressure increases during CPTu penetration.

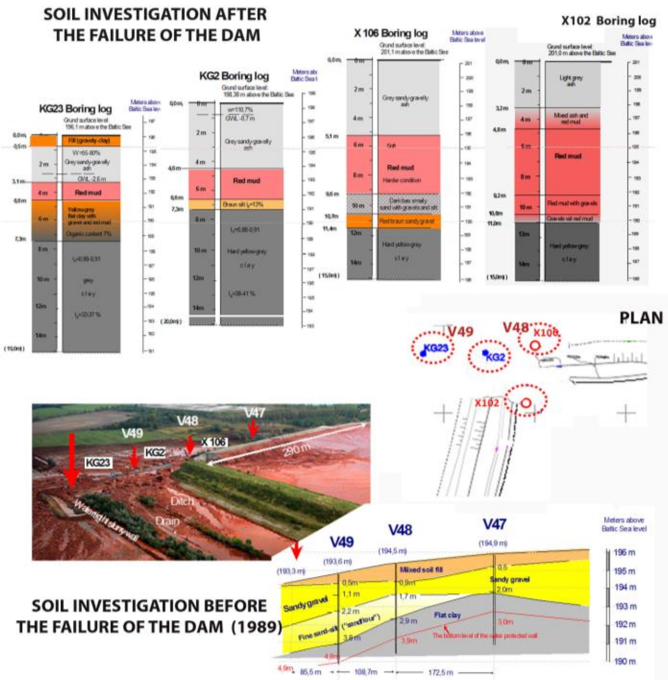

Fig. 14. Soil investigation results after the failure of the dam. Boring logs- photo and plan with the soil investigation places, and compare the soil profile before the failure

A clay-silt layer of very low resistance is located under the compacted gravel layer, which is extremely soft until a depth of 12 m, with a gradually increasing rigidness at larger depths. Pore water pressure in the clay layer is insignificant, which shows that it is the sandy gravel layer that was affected by an extremely high water pressure.

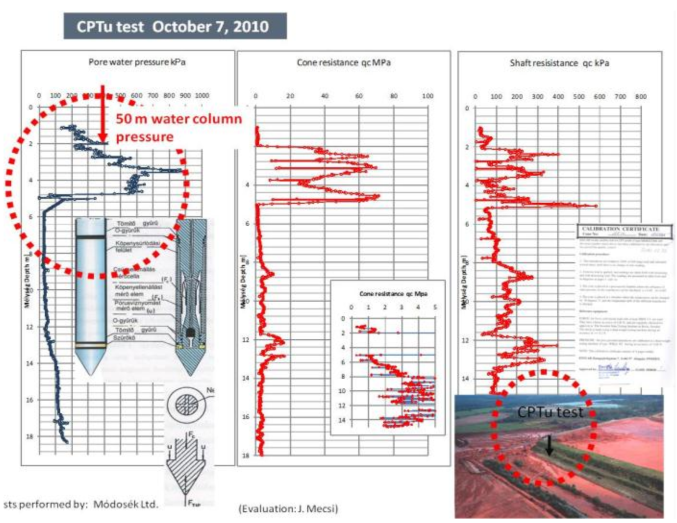

Fig. 15. CPT test results 3 days after the failure

It must be emphasized that although the sandy gravel–gravel soil layer of large bearing capacity is favorable in terms of potential subsidence, but at the same time it is unfavorable in terms of water pressures that may evolve in a system of communicating vessels.

5. ALTITUDE CONDITIONS

Figure 16 shows crown level heights of the embankments of Reservoir No. 10 in their condition before the embankment failure in October 2010. The boundary embankment is split into sections on the site plan and the altitude data are shown on a "developed" boundary line.

The figure shows rather significant distortions as horizontal and altitude scales largely differ from each other.

As it can be observed, the highest section of the dam crown is between points 'D' and 'E', i.e. at the embankment between Reservoirs No. 9 and 10, showing an altitude of 218,0-219 mBf (meters above Baltic Sea level).

The embankment at the northern side of Reservoir No. 10 is the lowest in altitude, sloping towards the west. The degree of slope is 1.3–1,5% (as the altitude difference of 1.2–1.4 m develops at a very long section).

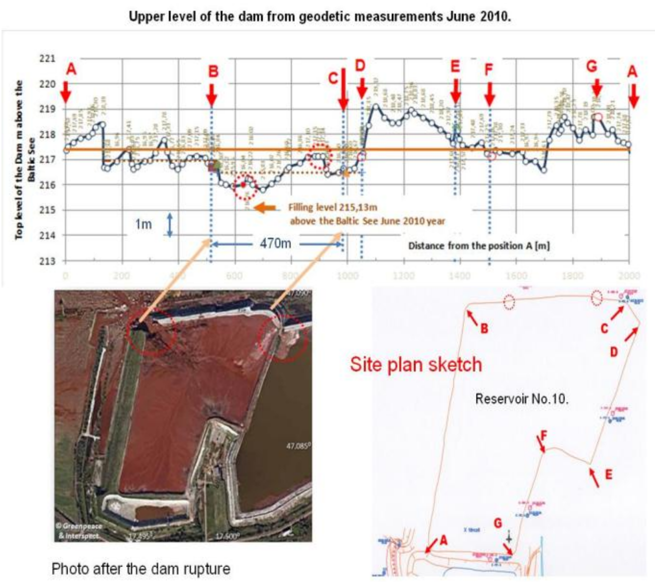

Fig. 16. Altitude data of the crown level of Reservoir No. 10

Fig. 17. Photo of Reservoir No. 10 when the filling of red mud into the reservoir was at an early stage

The average crown level height is 217,3 mBf. and the difference between the lowest and the highest crown levels is 2.5 m, which cannot be regarded as extreme considering the fact that the total length of the embankments is several kilometers.

It can also readily be assumed that this embankment section had been meant to be constructed lower at the initial construction as it had been considered a temporary embankment, as an extension in northern direction had been to be built later, which is clearly confirmed by the original plans. The original photo – in which the level of red mud filled in the reservoir indicates the date when the picture was taken – also clearly shows that the embankment was lower when originally built (Figure 17.).

Significant differences (4-6 m) is on the north side of the dam and the east side of the barrier thickness (25-27 m). The temporary dam was built on the north side, it is assumed that you have bought into the reservoir space for future expansion ideas as well.

6. THE CLOSURE OF THE RESERVOIR SYSTEM

Water pollution was detected in the groundwater monitoring wells near the reservoir system in the 1970s–1980s. In compliance with the regulatory requirements, a watertight slurry wall was constructed to close down the southern and the western sides of the reservoir. Later, as the pollution spread over towards the north, the construction of a new type of grout curtain was started around the reservoir system in 1999. The length of the grout curtain was 1045 m on the northern side, 200 m on south side, and there were connected to the already existing slurry wall. Figure 18 shows the boundary structure around the reservoir system. The depth of the vertical closure was determined in relation to the depth of the watertight clay substratum, by minimum 1.0 m bellow it. The depth of the southern wall varies between 3.0 and 8.0 m, while 6.0-9.0-12.0 m appear on the south-eastern side.

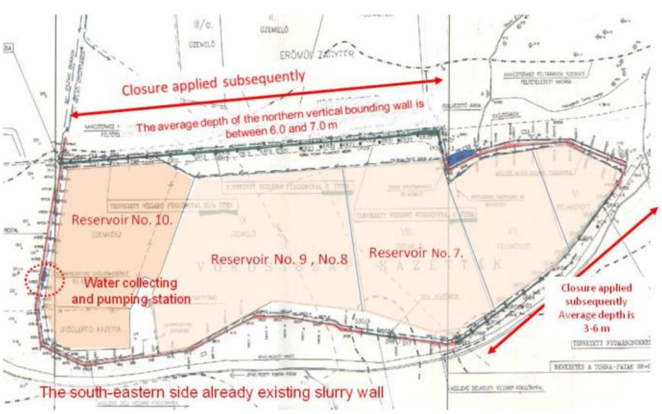

Fig. 18. Site plan of the slurry wall and grout observed in the 1998 year (curtain boundary constructed at several stages)

7. CHARACTERISTICS OF THE EMBANKMENT

The boundary embankments show some special features. As they were made of slag and ash from the power plant, their weight is relatively low. Their average density is ρ= 1.5-1.55 gr/cm^3, while their dry density is ρs= 0,7-0,8 gr/cm^3.The embankment is quasi-saturated with water. Due to the hydraulic chemical bond, the embankment is characterized by a relatively large strength. As a result of the construction technology applied, the embankment is layered and is of inhomogeneous structure, which manifests both in its strength and its water permeability characteristics. Figure 19 shows a characteristic profile of the northern embankment of Reservoir No. 10 after the failure.

Typical material tests were performed of the southern embankment by the Institute of Environmental Management of the University of Miskolc in November 2010. It must be noted that properties determined at the embankment of Reservoir No. 10/a may differ from the test results of the embankment of Reservoir No. 10, but in general, they give an approximate picture of the strength, water permeability and density properties of the embankment. The dam material is horizontally layered and is inhomogeneous in the sense that it is built up of layers of finer and coarser grain composition. The joint surfaces detected in core samples were located in the zones composed of coarser grains as the stability of zones with coarser grains is lower than that of layers built up of finer grains.

The value of unidirectional compressive strength of the coarse grain samples was 0.5-0.8 MPa at deeper sampling locations, while it was significantly lower, 0.2-0.45 MPa closer to the surface. In the case of fine grain samples the value of unidirectional compressive strength was usually between 1.5-2.5 MPa and in some extraordinary cases it reached as much as 2.8-2.9 MPa.

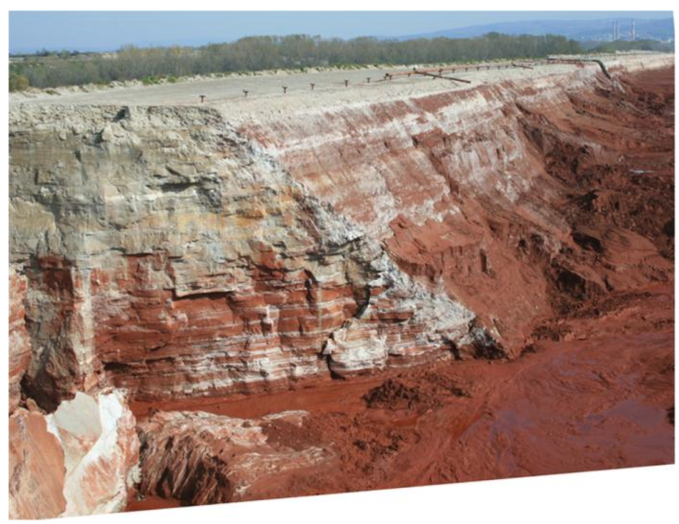

Fig. 19. Material of the northern embankment of Reservoir No. 10 after the embankment failure (photo: Dr. Tibor Horváth, November 2010)

Breaking strength values were around 4.5-6 kN at failures along the edge in the coarser and finer beds. In highly stratified zones composed of coarse grains, breaking force was as low as 2.5-4 kN, while in well preserved samples of the best bearing capacity, values of as much as 10 kN and in some cases 15 kN were observed.

In general, the core samples show good compressive and tensile strength in their natural condition, their unidirectional compressive strength being around 1.5 MPa, while in the case of failure along the edge the compressive strength at the centre line is calculated at 0.9 MPa and the tensile strength is of an average value of 0.3 MPa. The samples are of low hydraulic conductivity, falling in the range of mud–muddy clay–clay. Test results show that the typical seepage factor of the samples was within the range of 10^{-9}–10^{-8} m/s but in the case of a few samples values higher or lower than this range were measured, at 5,8•10-10 – $2{,}5{\cdot}10^{-7}$ m/s. In light of the above, the coarse-grain layers of low bearing capacity practically showed the seepage factor of a mud soil, while the more compact layers had that of fat clay. The impermeability of samples drops along with the decrease of grain size and thus along with the growth of total porosity. When a sudden increase of tension was exerted on the samples, they lost approximately third of their strength. However, no correlation was found between the size of the compression wave and the loss of strength within the range of 4-8 bars."

8. PROPERTIES OF THE "RED MUD"

"Red mud" is a by-product of alumina production, in the process of which bauxite is processed using the Bayer process. Bauxite is crushed and ground in mills and heated; alumina is precipitated by washing the bauxite with a hot solution of sodium hydroxide under pressure. "Red mud" is then gained from the remaining solution by settling and filtering.

The properties of "red mud" are reviewed drawing on past experience in a very informative article about the geotechnical aspects of hydraulic spoil banks (Asbóth, János – Szivák, Attila: Geotechnical aspects of hydraulic spoil banks, Civil Engineering Review, Volume XXXII, issue 12/1982, pp. 538-546). The statements of the article were later confirmed by the findings of tests performed later. „The Fe_2O_3 content of red mud is high and therefore material density is also higher than in average clays: it is ρs =2,9–3,3gr/cm^3. The value of water content varies within the range of w=150– 200 %. Bulk density is ρ=1,3–1,5gr/cm^3 in freshly settled clay and ρ= 1,5–1, 7 gr/cm^3 in clay settled for years.

Comparing the material density and the bulk density data it can be stated that the substance stored in the red mud reservoir is of extremely loose structure. In its fresh condition the void ratio of the clay varies between e = 4–6, while it is between 2–4 in spoil banks settled and abandoned for years."

The extraordinary volume changing character is shown by the fact that after the desiccation of its surface, cracks of 3-8 cm appear on it (Figure 20). In spite of the fact that the cracks penetrate into a depth of as much as a meter, the substance is characterized by a creamy consistency only a few millimeters under the surface.

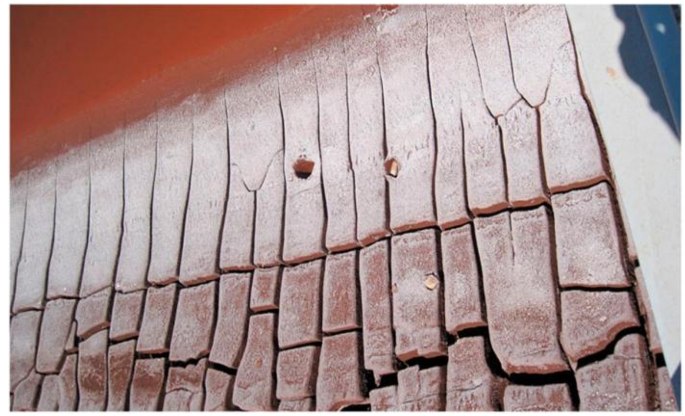

Fig. 20. The extraordinary volume changing character after the desiccation of red-mud surface, cracks of 3-8 cm appear on it (Surface of the "red mud" in dry condition (photo: J. Mecsi)

Corresponding liquid limit and plasticity index values in a Casagrande plasticity chart shows that the points plotted fall into the range of organic substances without exception. The description of the substance as „red mud" is not appropriate in terms of soil mechanics, as considering plasticity index, it belongs to the group of substances of medium to high plasticity according and should be described as medium and fat clay.

The following questions arise:

- What special characteristics determine the behaviour of the substance if it is defined as medium and fat clay, and it is to be regarded as a near-watertight material according to its permeability coefficient?
- Why can we not expect that the substance, as a watertight material, will prevent water seepage at the bottom of the reservoir and near its wall?

"Shear strength tests were performed with an airfoil shear probe. Examining the test results it can be stated that the authoritative shear strength is 10 kN/m^2 on the settling area currently in use, which approximately corresponds with the triaxial test results. Shear strength tests performed in the abandoned spoil bank showed more diffuse results than those in the currently used one, but the authoritative value was around 10 kN/m^2 there, too.

These tests also confirmed that "red mud" does not lose its water content and its shear strength does not increase along with the time passing."

Chemical effects, such as that of sodium hydroxide added to the swollen clay in the course of the technological process, may also have a role in the special behavior of "red mud".

It can conclusively be stated that the substance shows special thixotropic behavior, it does not easily lose its water content and assumes the behavior of a thick, plastic liquid upon significant loading.

9. PRECIPITATION DATA BETWEEN 2000 AND 2010

Fig.21 shows a graphic presentation of the precipitation data provided in a table by the Weather Data Services Department of the National Weather Service on October 8, 2010.

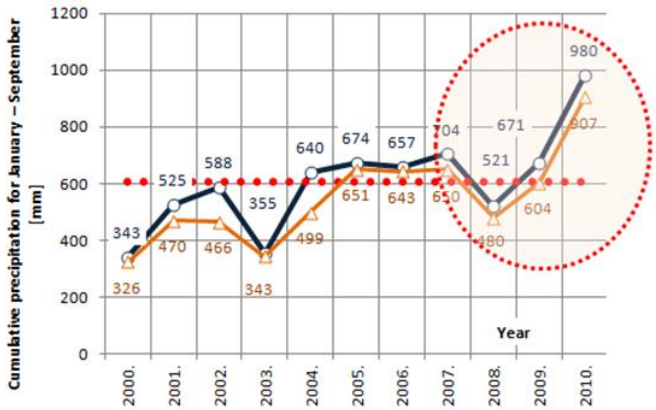

Fig. 21. Total precipitation data in the 1st– 3rd quarter year between 2000 and 2010. The figure shows data from two observation places near the investigated territory

Comparing the total precipitation of the period between January and the end of September 2010 with that of 2009, a 44% increase can be observed, while comparing the same with 2008 data, the increase in precipitation is 74%. This volume of water is rather significant considering the character of the drainage basin. The volume of total precipitation in the period between January and the end of September 2010 is more than one and a half times higher than the 10-year average (154%).

10. GROUNDWATER LEVEL AND WATER PRESSURE MONITORING DATA

The surface levels and the ground water levels observed in the area of the entire reservoir system. The initial water level at Reservoirs No. 6 and 7 was between 210-215 m. above the Baltic See (mBf.) according to the monitoring performed in 2011 (Budapest University of Technology, Geotechnical Department, March 2011), while the highest water pressure level observed at Reservoir No. 10 was 197,4 mBf on the northern side and 191,5 mBf on the southern side According to the above data, the difference in water pressure level is 15-17 m, with an additional 9.5 m between the northern and the southern parts of Reservoir No. 10. The above data were gained during a period of very low precipitation in 2011, while in 2010 precipitation was especially high. In light of the above, it can be concluded that the boundary walls constructed around the reservoir system due to environmental reasons in accordance with the requirements of the authorities had a very unfavorable effect, as a result of which the system could gather a very significant amount of precipitation in 2009-2010, which resulted in extremely high differences in water level and therefore extremely high differences in water pressure.

11. WIND SPEED AND WIND DIRECTIONS

The prevailing wind direction in the vicinity of Reservoir No. 10 is northern–north-western as shown in the wind direction frequency chart. Wind parallel with the direction of the embankment failure is very rare (Figure 22).

Peculiar wind conditions were observed in the area between September 25 and October 4. Between October 1 and 4, a gradually strengthening, unfavorable wind speed toward the direction of the embankment failure was accompanied by casual wind gusts (Weather data services, National weather Services, October 8, 2010).

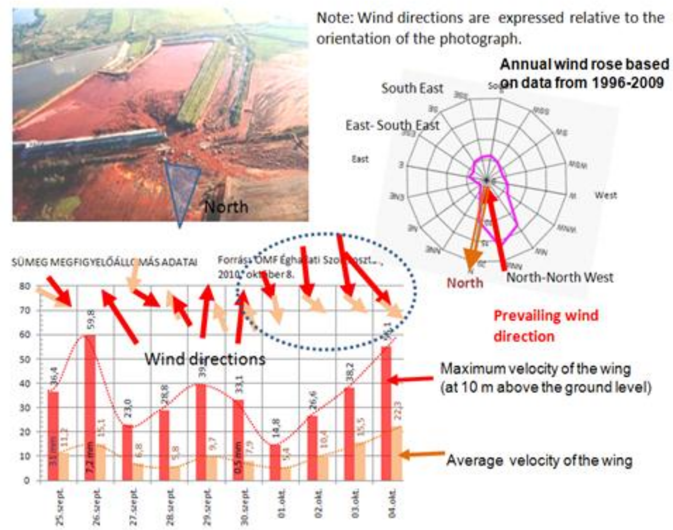

Fig. 22. Vectors of prevailing wind directions and wind speeds between September 25, 2010 and October 4, 2010 (Weather Data Services Department of the NWS)

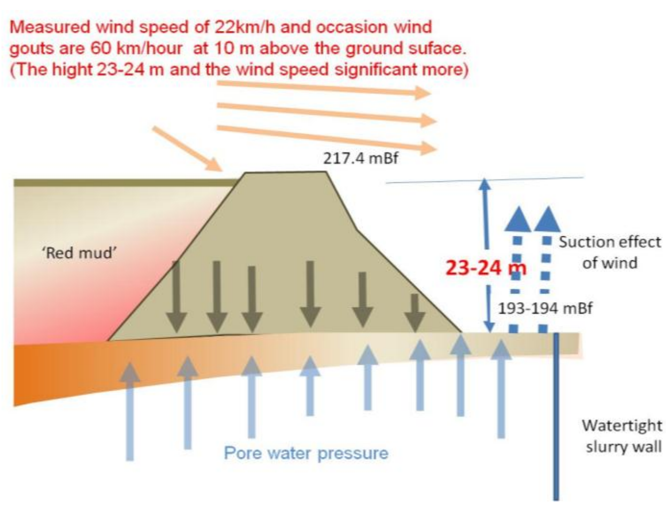

Fig. 23. Effects of the wind speeds between September 25 and October 4, 2010

The observations of the wind direction and velocity were at 10 m over the ground surface, but the top surface of the reservoir is 23-24m. It is mean that the wind velocity in site should be

significant more than the measured value. The highest wind speed values were measured on October 4, with an average of 22 km/h and occasion wind gusts of 60 km/h.
There is reason to assume that these unusual wind direction and wind speed conditions contributed to the development of the embankment failure. In addition, the possible sucking effect of the wind on the northern side of the embankment must also be taken into consideration. Fig. 23.

12. SATELLITE MONITORING DATA

Following the embankment failure, the Satellite Geodetic Observatory (KGO) of the Institute of Geodesy, Cartography and Remote Sensing (FÖMI) published its monitoring data in respect of Reservoir No. 10 on the internet. "InSAR study of the failed red mud reservoir near Kolontár, Figure 24.
It can be concluded from the monitoring data that the maximum range of the values measured was 1–1.3 cm/year and it showed a steady, gradually increasing tendency. At a few locations however a lower increase of a total value of 1.0–2 cm was measured within a period of 7 years.

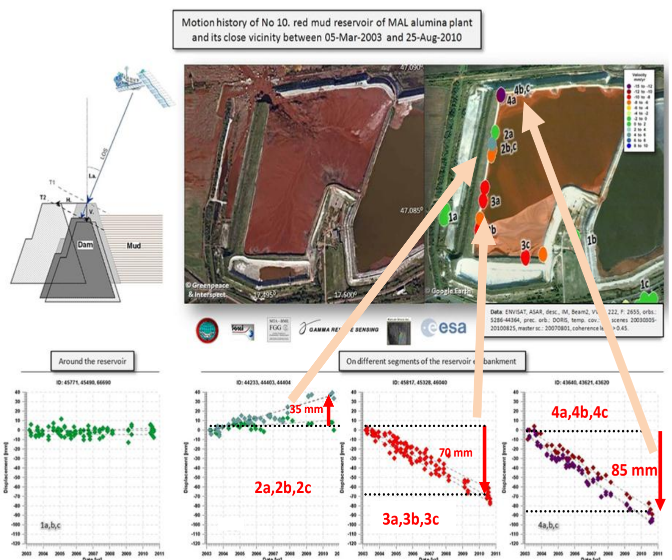

Fig. 24. Summary of the satellite monitoring data [DigitalGlobe, Sample Imagery Gallery, Earth explorer: U.S. Geological Survey http://edcsns17.cr.usgs.gov/EarthExplorer/ (2011), Satellite Geodetic Observatory (KGO) of the Institute of Geodesy, Cartography and Remote Sensing (FÖMI)]

The range of subsidence and movements is not significant but is indicative and can primarily originate from the "unsettled" behavior of the soil underlying the dam. Even considering measurement inaccuracy, the values measured cannot be regarded significant.
At locations 4a, b and c the subsidence of the embankment was 80 mm/7 years, i.e. 12-13 mm/year.
At locations 2a, b and c a slight rising of the embankment was experienced along a four-year period starting in 2003; the movement was later stabilized and the embankment occasionally rose but only to a very slight extent.
At locations 3a, b and c the total value of subsidence was 60 mm within a period of 7 years, which indicates an annual subsidence of 8 mm.

Monitoring data show a realistic range of values and further confirm the conclusions of the analysis. Furthermore, they verify the lengthy seepage process and washout of fine grains under the dam.

13. CIRCUMSTANCES AND COMPLEX CAUSES

The area is located in the lowest part of the drainage basin of the Torna stream, i.e. in the valley of the stream. In its original condition, the area, together with the subsurface stream valley formed a surface and subsurface water flow unit which was very sensitive to weather conditions. The area had been gradually involved in and shaped by industrial operations, through the construction of surface and subsurface structures. The northern, so called intermediate embankment of Reservoir No. 10, constructed in a smaller size with a view to the possible extension of the reservoir system, served as an external, i.e. boundary embankment. As a result, it was the surface run-off and flow conditions that were first changed significantly by the industrial use of the area in the stream valley.
It can be concluded that according to an engineering approach to finding the causes, the failure of the rigid embankment of large bearing capacity was a combined result of a number of unfavourable conditions.
The base width of the northern embankment is significantly different from the size of the other embankments, as it was considered a temporary structure, bearing in mind the possibility of a future extension. The slightly more than 20 m height of the solid part of the embankment is lower than the 26–27 m heights of the other embankments.

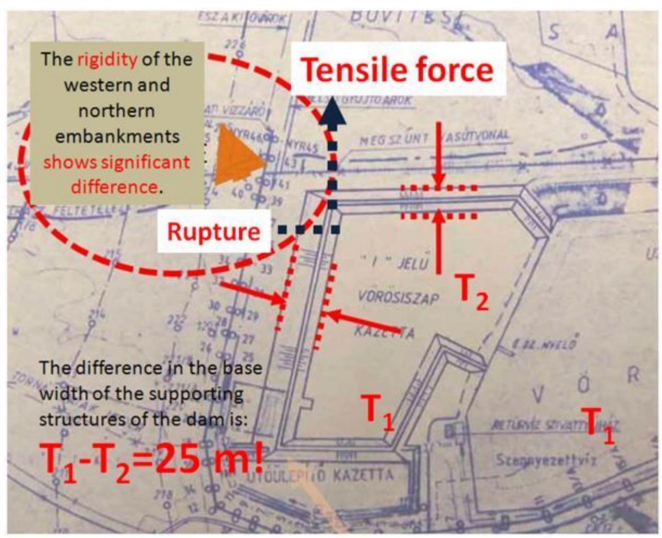

Fig. 25. Dimensions of the western and southern dams

Its effect manifested at the northern embankment – being by 25 m less in base width than the western embankment – as the resistance against displacement here was significantly lower, and also because the rigidity of the western and northern embankments showed significant difference at an unfavourable connection at the corner of the reservoir.

We concluded, that at the connection of the stronger west dam and softer north dam a very big tensile force could occur, while at the middle of the north dam big torsion force developed. Fig. 25. and Fig 26.

Fig. 26. The cracking at the middle of the north dam

The deformed, distorted northern dam after the catastrophic rupture, was not at rest.

Fissures started developing on the crown level of the northern dam section at a gradually increasing pace 3 days after the dam failure, following which the embankment subsided in 30-40 m pieces, with 20-30 cm settlements. Fig 27.

The leaking red mud stored in the reservoir left a crater by the inner side of the embankment, as a result of which the foot of the dam became exposed. (On in situ observations by J. Mecsi)

Fig. 27. Picture of the North dam of the reservoir No. 10 on October 7, 2011 based on in situ observations (photo: J. Mecsi)

Fig. 28. The damaged reservoir of an alumina plant is seen from the air near the village of Kolontar, Hungary, Monday, Oct. 11, 2010. AP / Abel Szalontai

The geotechnical conditions were determined by the fact that at the northern boundary embankment of the reservoir there are beds of easily liquefiable muddy fine sand (or sand flour as described earlier) in a layer of 3-4 m located in the closest vicinity of the critical corner of the reservoir, adjacent to a sandy-gravelly layer.

Significant pore water pressure may have developed in the soil layer underlying the embankment as a result of the geological properties of the enclosed gravel terrace functioning as a drainage basin and due to extremely high precipitation levels causing high water pressure conditions. Fig. 29.

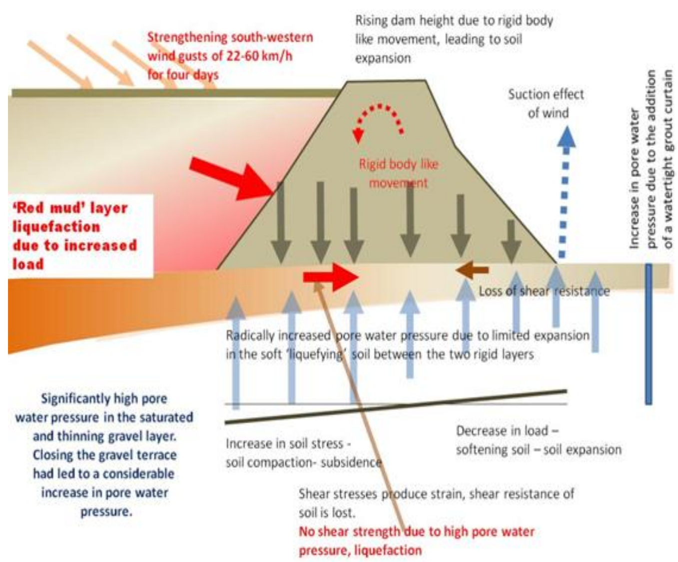

Fig. 29. Summarizing some effects for the dam failure

The increase of load on the slope surface of the embankment might have contributed to the excess load – and the increase of soil stress – in the subsoil on the inner side and as a consequence, to excess subsidence, while it probably caused a

slighter rate of expansion on the outer side due to the rigid body like movement of the embankment.
The uneven layer structure of the subsoil might have contributed to the possible development of torsion loading at the northern embankment, which is indicated by cracks.
The extremely rigid but light embankment and the soft, liquefiable subsoil provided highly unfavourable static conditions.
The liquefaction property and special thixotropic behavior of the "red mud" may have also contributed to the disaster. Chemical effects, such as that of sodium hydroxide added to the swollen clay in the course of the technological process, may also have a role in the special behavior of "red mud".
The extremely unfavorable wind direction and wind speed conditions may have given a final push toward the very sudden embankment failure.
It is also an important factor for the stability of the dam, that by what kind of filling technology is used. In what magnitude and distribution of water heights may occur within the area of the reservoir?
The list of contributing factors could be further extended and it will readily be conceived that it is the accumulation of unfavourable conditions that led to the sudden rupture of the embankment.

The present study does not aim to identify scapegoats of the incident. The findings and conclusions derived from the examinations may be further refined and supplemented in the future in light of further facts and data yet to be revealed.

14. SUMMARY AND CONCLUSIONS

The present study aims to provide a background to a non-exhaustive list of factors contributing to the embankment failure, while attempting to give a clear picture of the complex technical conditions.
The rupture of the embankment and the highly serious disaster emerging thereof serve as a lesson in several aspects for professionals performing technical or legislative tasks, as well as for those working in the area of the administration of justice and performing official control duties.
It is not an aim of the present study to identify scapegoats for the incident.

The findings and conclusions derived from the examinations may be further refined and supplemented in the future in light of further facts and data yet to be revealed.

The objective of the author of the present analysis is, led by deep sympathy for the victims and those who suffered damage, to provide an insight into the technical causes and the circumstances of the tragic incident, as well as to promote, with a humble approach to sciences, all endeavors to avoid such disasters in the future.

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